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Effect of Tuned Mass Dampers on Multistorey RC Framed Structures

Raveesh R M
M.Tech student, Department of Civil Engineering,
Sri Siddhartha Academy Of Higher Education,
Tumkur, India.

Sahana T S
Asst. professor, Department of Civil Engineering,
Sri Siddhartha Academy Of Higher Education,
Tumkur, India.

Abstract:- Earthquakes create vibrations on the ground that are translated into dynamic loads which cause the ground and anything attached to it vibrate in a complex manner and cause damage to buildings and other structures. Civil engineering is continuously improving ways to cope with this inherent phenomenon. Conventional strategies of strengthening the system consume more materials and energy. Moreover, higher masses lead to higher seismic forces. Alternative strategies such as passive control systems are found to be effective in reducing the seismic and other dynamic effects on civil engineering structures. The main focus of the present investigation is to evaluate effect of tuned mass dampers on the structural response of multistorey RC frame structures subjected to implemental dynamic analysis. A multistorey RC frame structure buildings having a ratio of height to breadth from 1, 2 and 3 is used in this study. The models were used to represent buildings located in zone 5 of India. The systemic parameters studied are natural time period, base shear, roof displacement, lateral displacement. A single ground motions were used in the study to generate single record Time v/s Acceleration curves namely BHUJ EARTHQUAKE. These ground motions was scaled to the design spectral acceleration prior to the application. The effect of acceleration is examined in this analysis SAP 2000, a program capable of performing nonlinear dynamic analysis. Based on the analysis results, it has been concluded that the effect of tuned mass dampers plays a significant role to decrease the natural frequency, base shear, roof displacement, lateral displacement, story drift, bending moment and shear force in a multistorey RC framed Structure.

Keywords: tuned mass dampers; multistorey RC framed structures; finite element method; SAP 2000; natural time period.

1. INTRODUCTION

Earthquakes are perhaps the most unpredictable and devastating of all natural disasters. They not only cause great destruction in terms of human casualties, but also have a tremendous economic impact on the affected area. An earthquake may be defined as a wave like motion generated by forces in constant turmoil under the surface layer of the earth (lithosphere), travelling through the earth's crust. It may also be defined as the vibration, sometimes violent, of the earth's surface as a result of a release of energy in the earth's crust. This release of energy can cause by sudden dislocations of segments of the crust, volcanic eruption, or even explosion created by humans. Dislocations of crust segments, however, lead to the most destructive quakes. In the process of dislocation, vibrations called seismic waves are generated. These waves travel outward from the source of the earthquake at varying speed, causing the earth to quiver or ring like a bell or tuning fork.

In every field in the world conservation of energy is followed. If the energy imposed on the structure by wind and earthquake load is fully dissipated in some way the structure will vibrate less. Every structure naturally releases some energy through various mechanisms such as internal stressing, rubbing, and plastic deformation. In large modern structures, the total damping is almost 5% of the critical. So new generation high rise building is equipped with artificial damping device for vibration control through energy dissipation. The various vibration control methods include passive, active, semi-active, hybrid. Various factors that affect the selection of a particular type of vibration control device are efficiency, compactness and weight, capital cost, operating cost, maintenance requirements and safety.

Tuned mass dampers (TMD) have been widely used for vibration control in mechanical engineering systems. In recent years, TMD theory has been adopted to reduce vibrations of tall buildings and other civil engineering structures. Dynamic absorbers and tuned mass dampers are the realizations of tuned absorbers and tuned dampers for structural vibration control applications. The inertial, resilient, and dissipative elements in such devices are: mass, spring and dashpot (or material damping) for linear applications and their rotary counterparts in rotational applications. Depending on the application, these devices are sized from a few ounces (grams) to many tons. Other configurations such as pendulum absorbers/dampers, and sloshing liquid absorbers/dampers have also been realized for vibration mitigation applications.

TMD is attached to a structure in order to reduce the dynamic response of the structure. The frequency of the damper is tuned to a particular structural frequency so that when that frequency is excited, the damper will resonate out of phase with the structural motion. The mass is usually attached to the building via a spring-dashpot system and energy is dissipated by the dashpot as relative motion develops between the mass and the structure.

1.1 Methods of Control.

A large numbers of technique have been tried to produce better control against wind and earthquake excitation. These can be classified into four broad categories: passive control, active control, semi-active control and hybrid control. Each of these will be discussed in following section.

1.1.1 Passive Control

Passive control devices are systems which do not require an external power source. These devices impart forces that are developed in response to the motion of the structure, for e.g., base isolation, viscoelastic dampers, tuned mass dampers, etc.

1.1.2 Active Control

Active control systems are driven by an externally applied force which tends to oppose the unwanted vibrations. The control force is generated depending on the feedback of the structural response. Examples of such systems include active mass dampers (AMDs), active tendon systems, etc.

1.1.3 Semi-active control

Semi-active systems are viewed as controllable devices, with energy requirements orders of magnitude less than typical active control systems. These systems do not impart energy into the system and thus maintain stability at all times, for e.g., variable orifice dampers, electro-rheological dampers, etc

1.1.4 Hybrid control

Hybrid systems act on the combined use of passive and active control system. For example, a base isolated structure which is equipped with actuator which actively controls the enhancement of its performance.

1.2 Types of passive control devices.

1.2.1 Metallic yield dampers

One of the effective mechanisms available for the dissipation of energy, input to a structure from an earthquake is through inelastic deformation of metals. The idea of using metallic energy dissipators within a structure to absorb a large portion of the seismic energy began with the conceptual and experimental work of Kelly et al. (1972) and Skinner et al. (1975). Several of the devices considered include torsional beams, flexural beams, and V-strip energy dissipators. Many of these devices use mild steel plates with triangular or hourglass shapes so that yielding is spread almost uniformly throughout the material.

1.2.2 Friction damper

Friction provides another excellent mechanism for energy dissipation, and has been used for many years in automotive brakes to dissipate kinetic energy of motion. In the development of friction dampers, it is important to minimize stick-slip phenomena to avoid introducing high frequency excitation. Furthermore, compatible materials must be employed to maintain a consistent coefficient of friction over the intended life of the device.

1.2.3 Viscoelastic dampers

The metallic and frictional devices described are primarily intended for seismic application. But, viscoelastic dampers find application in both wind and seismic application. Their application in civil engineering structures began in 1969 when approximately 10,000 visco-elastic dampers were installed in each of the twin towers of the World Trade Center in New York to reduce wind-induced vibrations. Further studies on the dynamic response of viscoelastic dampers have been carried out, and the results show that they can also be effectively used in reducing structural response due to large range of intensity levels of earthquake. Viscoelastic materials used in civil engineering structure are typical copolymers or glassy substances.

1.2.4 Viscous fluid dampers

Fluids can also be used to dissipate energy and numerous device configurations and materials have been proposed. Viscous fluid dampers, are widely used in aerospace and military applications, and have recently been adapted for structural applications (Constantinouet al. 1993). Characteristics of these devices which are of primary interest in structural applications, are the linear viscous response achieved over a broad frequency range, insensitivity to temperature, and compactness in comparison to stroke and output force. The viscous nature of the device is obtained through the use of specially configured orifices, and is responsible for generating damper forces that are out of phase with displacement.

1.2.5 Tuned liquid damper

A Tuned liquid damper (TLD)/tuned sloshing damper (TSD) consists of a tank partially filled with liquid. Like a TMD, it imparts indirect damping to the structure, thereby reducing response. The energy dissipation occurs through various mechanisms: viscous action of the fluid, wave breaking, contamination of the free surface with beads, and container geometry and roughness. Unlike a TMD, however, a TSD has an amplitude dependent transfer function which is complicated by nonlinear liquid sloshing and wave breaking.

1.2.6 Tuned mass dampers

The concept of the tuned mass damper (TMD) dates back to the 1940s (Den Hartog 1947). It consists of a secondary mass with properly tuned spring and damping elements, providing a frequency-dependent hysteresis that increases damping in the primary structure. The success of such a system in reducing wind-excited structural vibrations is now well established. Recently, numerical and experimental studies have been carried out on the effectiveness of TMDs in reducing seismic response of structures (for instance, Villaverde (1994).

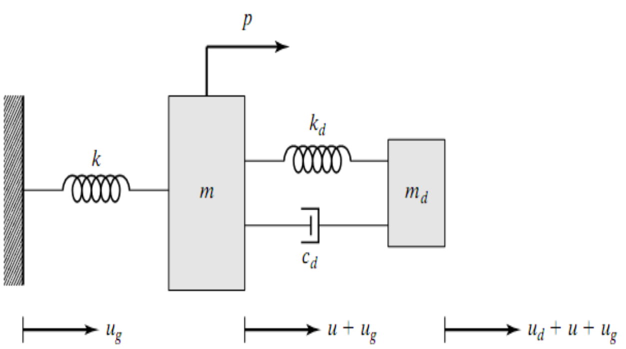
Tuned mass damper is attached to a vibrating structure to reduce undesirable vibrations. Tuned mass damper is a passive energy absorbing device consisting of a mass, a spring and a viscous damper. The mass is usually attached to the building via a spring-dashpot system and energy is dissipated by the dashpot as relative motion develops between the mass and the structure. The frequency of the damper is tuned to a particular structural frequency so that when that frequency is excited, the damper will resonate out of phase with the structural motion. Till date tuned mass damper have been installed in large number of structures all around the globe. The first structure in which tuned mass damper was installed is the Centre point Tower in Sydney, Australia. There are two buildings in the United States equipped with tuned mass dampers; one is the Citicorp Centre in New York City and the other is the John Hancock Tower in Boston. Chiba Port Tower (completed in 1986) was the first tower in Japan to be equipped with a tuned mass damper. In Japan, countermeasures against traffic-induced vibration were carried out for two two-story steel buildings under an urban expressway viaduct by means of tuned mass dampers (Inoue et al.1994). Results show that peak of the acceleration response of the two buildings were reduced by about 71% and 64%, respectively, by using the tuned mass dampers with the mass ratio about 1%.

Tuned mass damper in Taipei

2. METHODOLOGY

In the present analysis, a finite element method of analysis has been adopted, because of its diversity & flexibility as an analysis tool, with number of software tools available. In the present study, the structure is modeled as a 3-dimensional RC frame using software package SAP2000 (version-14.2.4). This minimizes the numerical calculations and also verification of results. Linear time history analysis can be performed on three dimensional structural models.

2.1 *TUNED MASS DAMPER THEORY*



The equations of motion for this case are

The inclusion of the damping terms in Eqn (6) and (7) produces a phase shift between the periodic excitation and the response. So, it is convenient to consider the solution expressed in terms of complex quantities. Then the excitation is expressed as

Where, $\hat{a}g$ and $\hat{p}$ are real quantities.

Then the response is given by

Where the response amplitudes, u and ud, are considered as complex quantities. Substituting Eqn (8) and (9) in the equations (6) and (7) and cancelling $e^{i\Omega t}$ from both sides results in the following equations

$[-m_d\Omega^2 + ic_d\Omega + k_d]\,\bar{u}_d - m_d\Omega^2\,\bar{u} = -m_d\,\hat{a}_g$

$-[ic_d\Omega + k_d]\,\bar{u}_d + [-m\Omega^2 + k]\,\bar{u} = -m\hat{a}_g + \hat{p}$

The solution of the governing equations is

$\bar{u}_d = (\hat{p}\rho^2/k\, D_2) - (\hat{a}_g m/\, k\, D_2)$

Where

$D_2 = [1-\rho^2][f^2-\rho^2] - \bar{m}\rho^2 f^2 + i^2\,\xi_d \rho f[1-\rho^2(1+\bar{m})]$

$f = \frac{wd}{w}$

Converting the complex solutions to polar form leads to the following expressions:

$\bar{u} = \frac{\hat{p}}{k} H_1 e^{i\delta 1} - (\hat{a}_g m/k)\, H_2 e^{i\delta 2}$

$\bar{u}_d = \frac{\hat{p}}{k}\, H_3\, e^{-i\delta 3} - (\hat{a}_g m/k)\, H_4 e^{-i\delta 4}$

where the H factors define the amplification of the pseudo-static responses, and the δs are the phase angles between the response and the excitation. The various H terms are as follows

$H_1 = [f_2-\rho_2]^2 + [2\varepsilon_d \rho f]^2) /|D2|$

$H_2 = (1+\bar{m})f^2 - \rho^2] + [2\,\epsilon_d \rho f\,(1+\bar{m})]^2$

$H3 = \rho 2/|D2|$

$H4 = 1/|D2|$

$$|D2| = ([1-\rho^2][f^2-\rho^2] - \overline{m}\rho^2 f^2)^2 + (2\epsilon_d \rho f[1-\rho^2(1+\overline{m})])^2$$

2.2 *TMD design for damped structure.*

An MDOF linear structure defined by the mass matrix **M** and the stiffness matrix **K** shall be damped in a specific mode with the mode shape ϕ and the angular eigen frequency $\boldsymbol{\omega_0}$ by means of a tuned mass damper with the mass $\mathbf{m_d}$, the damper constant $\mathbf{c_d}$ and the spring stiffness $\mathbf{k_d}$. The damper is acting co-directional to the j^{th} degree of freedom with the mode shape component $\boldsymbol{\phi_j}$.
At the design of the tuned mass damper, i.e. the determination of M_d, C_d and K_d, the following steps are followed:

1. Determine the mass and stiffness parameters M and K of the primary system.

$$M = \frac{\Phi^T M \Phi}{\Phi_j^2} \quad , \quad K = \omega_0^2 M$$

2. Specify the required modal damping ς of the considered mode of the primary system.

3. Calculate the damping ratio of the secondary system

$$\zeta_d = 2\zeta$$

4. Calculate the mass ratio

$$\mu = \frac{2\zeta_d^2}{1-2\zeta_d^2}$$

5. Calculate the angular frequency of the secondary system

$$\omega_d = \frac{1}{1+\mu}\omega_0$$

6. Calculate m_d, c_d and k_d

$$m_d = \mu M$$
$$k_d = \omega_d^2 m_d$$
$$c_d = 2\sqrt{k_d m_d}\,\zeta_d$$

3 PRESENT STUDY

In this study, multi-storey RC framed buildings with and without tuned mass dampers is considered. The time history analysis of these buildings has been done by subjecting the whole system to earthquake ground motions, using SAP-2000. The influence of tuned mass damper due to real time seismic excitations is studied in detail for multi-story RC structures and also variation in number of storeys.

The following studies have been carried out in the project work.

- A multi-story concrete structure having a ratio of breadth to height from1, 2 and 3 is used in the present study. The models were used to represent buildings located in zone 5 of India. The systemic parameters studied are the natural time period, base shear, lateral displacement and roof displacement.
- A setoff single recorded time v/s acceleration curves is obtained for each systemic parameters. A single ground motions were used in the study to generate single record time v/s acceleration curves for BHUJ EARTHQUAKE. These ground motions was scaled to the design spectral acceleration prior to the application. The effect of acceleration is examined in this analysis SAP 2000, a program capable of performing nonlinear dynamic analysis.

3.1 Input Parameters

A three dimensional five bay RC Frame structure models with Ten, Twenty and Thirty storey with fixed base, without and with tuned mass dampers of different mass ratio 0.25, 0.5 and 0.75 are considered.

Table 3.1: Range of parameters considered in the present study

Structure Type	Ordinary moment resisting frame
No. of storey	G+9, G+19, G+29
Typical storey height	3.0 m
Type of building use	Public building
Seismic zone	V
Soil type	Medium
Material Properties	
Grade of concrete	M25
Young's modulus of concrete, E_c	25x 10^6 kN/m^2
Grade of steel	Fe 415
Density of reinforced concrete	25 kN/m^3
Poisson's Ratio of reinforced concrete	0.20
Modulus of elasticity of brick masonry	1000 x 10^5 kN/m^2
Density of brick masonry	20 kN/m^3
Poisson's Ratio of brick masonry	0.15
Material Properties	
Thickness of slab	0.150 m
Beam size 0.230 x 0.600 m	0.300 x 0.500 m
Column size (10-storeyed building)	0.300 x 0.600 m
Column size (20-storeyed building)	0.300 x 0.900 m
Column size (30-storeyed building)	0.300 x 1.200 m
Thickness of external masonry wall	0.230 m
Thickness of concrete shear wall	0.230 m
Dead Load Intensities	
Roof finishes	2.0 kN/m^2
Floor finishes	1.0 kN/m^2
Partition wall load	1.0 kN/m^2
Live Load Intensities	
Roof	1.5 kN/m^2
Floor	3.0 kN/m^2

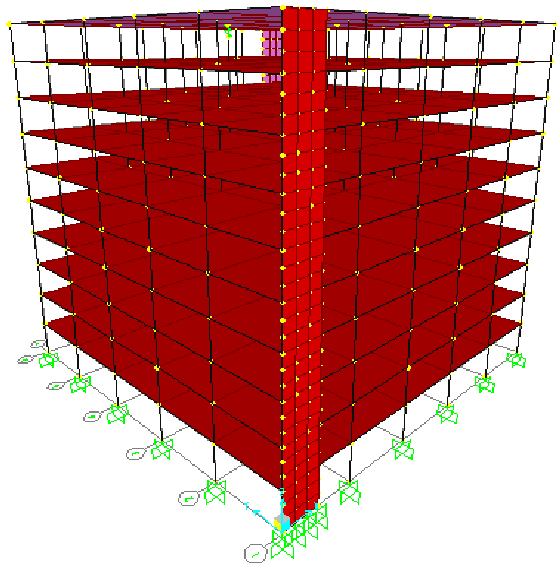


Fig 3.2 3D model of a 10 storey building considered for present study

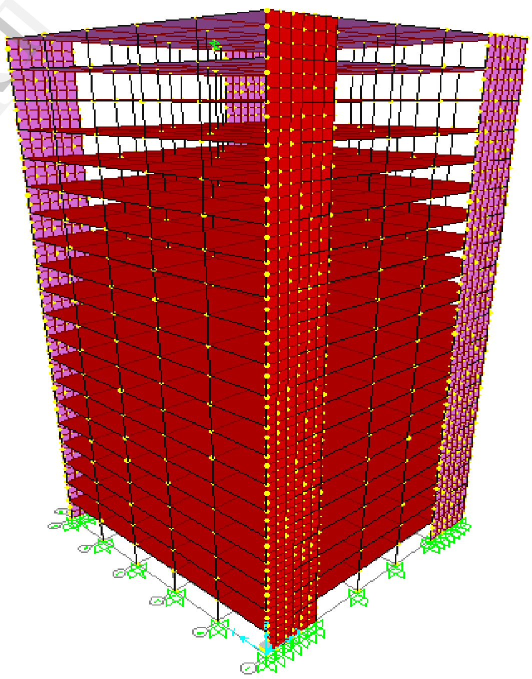


Fig 3.3 3D model of a 20 storey building considered for present study

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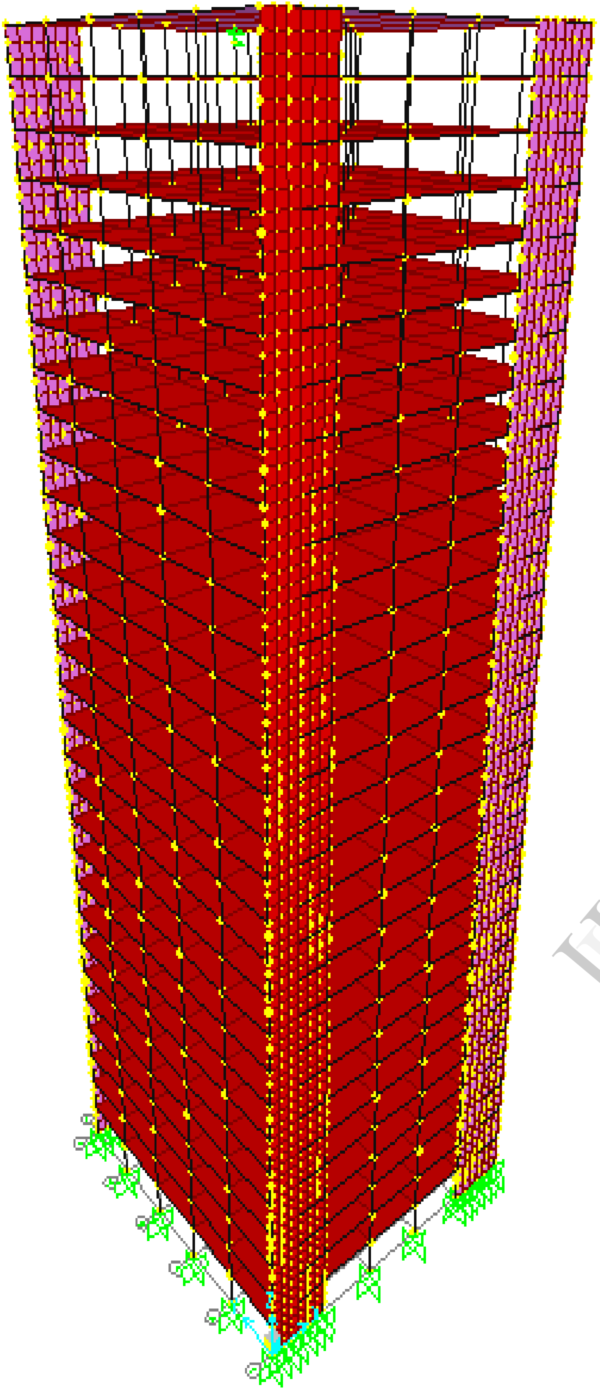


Fig 3.4 3D model of a 30 storey building considered for present study.

4 .RESULTS AND DISCUSSIONS

In the present study a three dimensional five bay RC Frame structure models with Ten, Twenty and Thirty storey with fixed base, without and with tuned mass dampers of different mass ratio 0.25, 0.50 and 0.75 subjected to acceleration v/s time history of BHUJ earthquake is studied. The tuned mass damper is modeled with elastic spring property. The variation of natural period and structural response for various parameters like base shear, roof displacements, and lateral displacements is presented.

.4.1 Variation in natural time period

The variation in natural time period due to the effect of tuned mass damper is studied on five-bay RC building modeled with height to breadth ratio 1, 2 and 3 considering base is fixed. The tuned mass damper is modeled with different mass ratio 0.25, 0.5 and 0.75 the results are tabulated in tables 4.1, 4.2 and 4.3. The mode shapes of 10, 20 and 30 storey building model with different mass ratio is shown in the Fig 4.1 to Fig 4.4.

4.1.1 EFFECT OF MASS RATIO

In this discussion comparison is carried out for each model with different mass ratios.

It is observed here that, natural period increases as the mass ratio increases and the percentage variation of natural period increases with increase in number of storey.

The percentage variations for different mass ratio with different number of storeys are tabulated in the Table 4.1 to Table 4.3.

Table 4.1 Values of natural period for 10 storey with and without TMD of different mass ratios.

COMPARISION OF NATURAL PERIOD FOR 10 STOREY BUILDING				
No of modes	Without TMD	M_d/M=0.25	M_d/M=0.50	M_d/M=0.75
Mode 1	1.0370	1.2862	1.5181	1.7085
Mode 2	0.6889	0.8357	1.0072	1.1338
Mode 3	0.6465	0.6918	0.6946	0.6969

The results obtained for Modal Period is shown in the Table 4.1 for 10 storey building. Introduction of TMD in RC frame increases the time period of RC frame without TMD. It has been found that TMD of mass ratio Md/M 0.75 configuration there was 39.30% increase in time period compared to RC frame without TMD for Mode 1. And in other for mass ratios Md/M (i.e. 0.25,0.5 and 0.75) models 17.57%, 31.60%and 39.24% increases in modal period was observed compared to RC frame without TMD for Mode 2.

Table 4.2 Values of natural period for 20 storey with and without TMD of different mass ratios.

COMPARISION OF NATURAL PERIOD FOR 20 STOREY BUILDING				
No of modes	Without TMD	M_d/M=0.25	M_d/M=0.50	M_d/M=0.75
Mode 1	1.6979	2.1486	2.3670	3.2495
Mode 2	1.2091	1.5270	1.8740	2.4331
Mode 3	1.1712	1.1792	1.1844	1.3689

The results obtained for Modal Period is shown in the Table 4.2 for 20 storey building. Introduction of TMD in RC frame increases the time period of RC frame without TMD. It has been found that TMD of mass ratio M_d/M 0.75 configuration there was 50.30% increase in time period compared to RC frame without TMD for Mode 2. And in other for mass ratios Md/M (i.e. 0.25,0.5 and 0.75) models 20.46%, 28.29% and 47.75% increases in modal period was observed compared to RC frame without TMD for Mode 1.

Table 4.3 Values of natural period for 30 storey with and without TMD of different mass ratios.

COMPARISION OF NATURAL PERIOD FOR 30 STOREY BUILDING				
No of modes	Without TMD	M_d/M=0.25	M_d/M=0.50	M_d/M=0.75
Mode 1	2.5723	3.0489	3.4968	3.9138
Mode 2	2.0498	2.5778	3.0318	3.4301
Mode 3	1.9725	1.9864	1.9907	1.9931

The results obtained for Modal Period is shown in the Table 4.3 for 30 storey building. Introduction of TMD in RC frame increases the time period of RC frame without TMD. It has been found that TMD of mass ratio Md/M 0.75 configuration there was 40.24% increase in time period compared to RC frame without TMD for Mode 2. And in other for mass ratios Md/M (i.e. 0.25,0.5 and 0.75) models 0.69%, 0.91% and 1.03% increases in modal period was observed compared to RC frame without TMD for Mode 3.

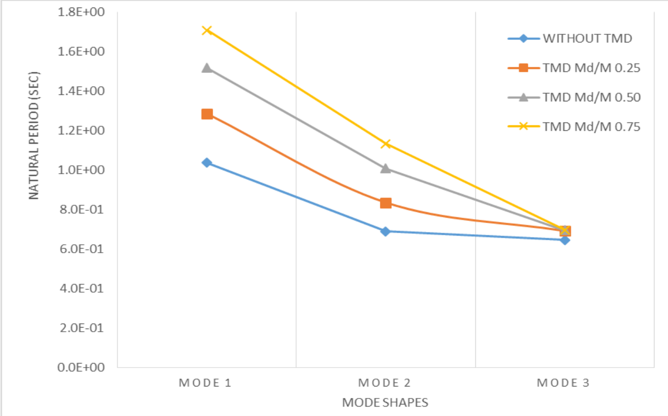
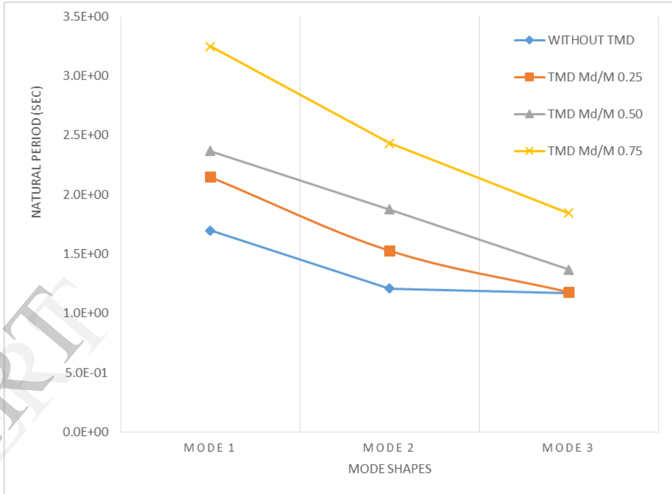


Fig 4.1 Variation of Time Period of 10 Storey Building with different Mass Ratio.



4.2 Variation of Time Period of 20 Storey Building with different Mass Ratio.

Fig 4.3 Variation of Time Period of 30 Storey Building with different Mass Ratio

From the Fig 4.1, 4.2and 4.3 it can be seen that for the increase in mass ratio there is increase in natural period for Mode 1, Mode 2 and Mode 3.so as the natural period increases the natural frequency of a building decreases so that the building is less vulnerable to earthquake.

4.2 VARIATION IN STRUCTURAL RESPONSE:

The seismic structural responses like the base shear, roof displacement, lateral displacement, storey drift, shear force, and bending moment for ground floor corner column of 10, 20 and 30 storey buildings are studied. For a BHUJ earthquake ground motion, the effect is studied by considering with and without TMD with varying mass ratios 0.25, 0.50 and 0.75 the results are presented in the following tables and the maximum variation of structural response quantities are plotted. The time history of structural response quantities like base shear and roof displacement are also plotted.

4.2.1 VARIATION IN BASE SHEAR:

Variation in base shear due to earthquake motions for all building frames with different mass ratios TMD are studied. The values of base shear are given in the tables 4.4, 4.5 and 4.6

Table 4.4 Values of Base Shear (10 storey)

COMPARISION OF BASE SHEAR FOR 10 STOREY BUILDING				
BASE SHEAR(KN)	Without TMD	M_d/M=0.25	M_d/M=0.50	M_d/M=0.75
	250.5	143.7	150.1	158.2

It is observed that for 10 storey building there is a substantial decrease in base shear during the period of strong shaking in the case of Md/M 0.25, 0.5 and 0.75 as compared to without TMD building for the ground motion BHUJ. There is a decrease of 42.63% in comparison with without TMD is seen in base shear for mass ratio Md/M 0.25 and 40.07% for mass ratio Md/M 0.50.

Table 4.5 Values of Base Shear (20 storey)

COMPARISION OF BASE SHEAR FOR 20 STOREY BUILDING				
BASE SHEAR(KN)	Without TMD	M_d/M=0.25	M_d/M=0.50	M_d/M=0.75
	307.1	176.0	212.9	289.1

It is observed that for 20 storey building there is a substantial decrease in base shear during the period of strong shaking in the case of Md/M 0.25, 0.50 and 0.75. As compared to without TMD building for the ground motion BHUJ. There is a 42.69% decrease in base shear when compared with without TMD for mass ratio Md/M 0.25 and 30.67% for mass ratio Md/M 0.50.

Table 4.6 Values of Base Shear (30 storey)

COMPARISION OF BASE SHEAR FOR 30 STOREY BUILDING				
BASE SHEAR(KN)	Without TMD	M_d/M=0.25	M_d/M=0.50	M_d/M=0.75
	449.8	292.0	336.2	427.1

It is observed that for 30 storey building there is a substantial increase in base shear during the period of strong shaking in the case of without TMD building for the ground motion BHUJ. After the implementation of TMD there is a 35.05% decrease in base shear when compared to without TMD is seen for mass ratio Md/M 0.25 and 25.26% for mass ratio Md/M 0.50.

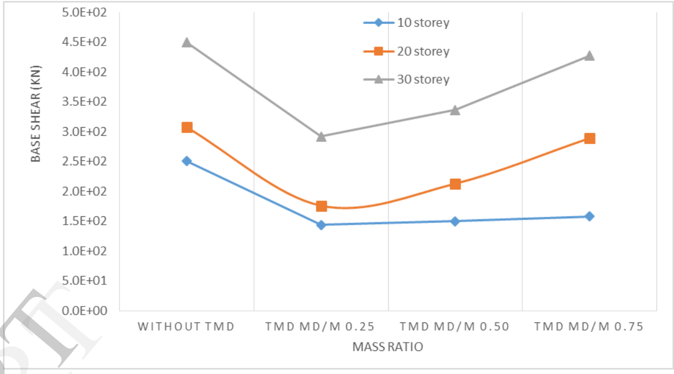


Fig 4.4 Variation of Base Shear of a 10, 20 and 30 storey building with different Mass Ratios.

The time history of base shear for various building models with and without TMD are plotted in fig 4.5, 4.6 and 4.7. It is seen from the time history curves that for ten storey building model without TMD under BHUJ motion, the base shear peak amplitude is occurring at 26 sec. When TMD is incorporated in the building model with different mass ratio 0.25, 0.5 and 0.75, the base shear peak amplitude is occurring at 32 sec, 31.2 sec and 27.4 sec respectively. The peak acceleration for a BHUJ ground motion is 29.46 sec, the peak acceleration time of a building increases with decrease in Mass Ratio of TMD.

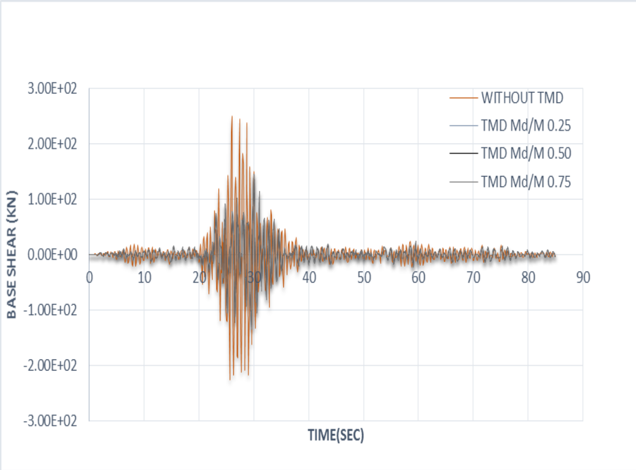


Fig 4.5 Variation of Base Shear of a 10 storey building with and without TMD under different Mass Ratios.

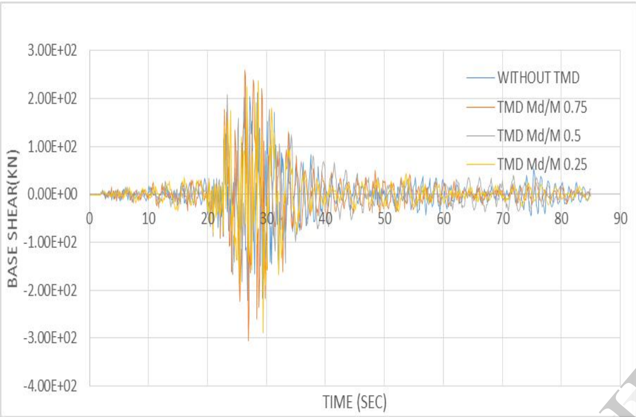


Fig 4.6 Variation of Base Shear of a 20 storey building with and without TMD under different Mass Ratios.

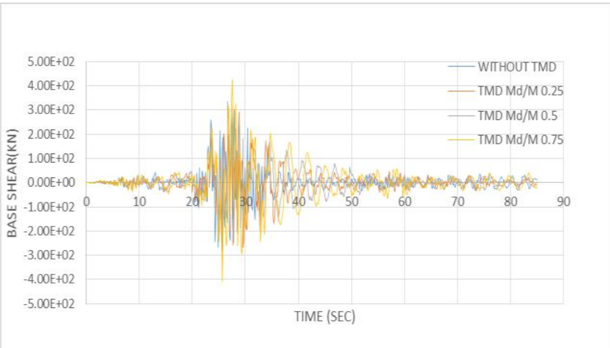


Fig 4.7 Variation of Base Shear of a 30 storey building with and without TMD under different Mass Ratios.

5.2.3 VARIATION IN LATERAL DISPLACEMENT:

Variation in Lateral Displacement due to earthquake motions for all building frames with different mass ratios TMD are studied. The values of Lateral displacement are given in the tables 4.7, 4.8 and 4.9.

Table 4.7 Values of Lateral Displacement (10 storey)

COMPARISION OF LATERAL DISPLACEMENT FOR 10 STOREY BUILDING				
lateral displacement at top of a storey along X(mm)	Without TMD	M_d/M=0.25	M_d/M=0.50	M_d/M=0.75
	20.168	12.671	16.398	18.476

From the table it can be concluded that the for 10 storey building there is a substantial decrease in lateral displacement during the period of strong shaking in the case of Md/M 0.25, 0.5 and 0.75. As compared to without TMD building for the Earthquake ground motion. There is variation of 37.17% decrease in lateral displacement compared with without TMD is seen for mass ratio Md/M 0.25, 18.96% for mass ratio Md/M 0.5 and 8.39% for mass ratio Md/M 0.75.

Table 4.8 Values of Lateral Displacement (20 storey)

COMPARISION OF LATERAL DISPLACEMENT FOR 20 STOREY BUILDING				
lateral displacement at top of a storey along X(mm)	Without TMD	M_d/M=0.25	M_d/M=0.50	M_d/M=0.75
	40.192	22.168	26.568	31.808

From the table it can be concluded that the for 20 storey building there is a substantial increase in lateral displacement during the period of strong shaking in the case of Md/M 0.25, 0.5 and 0.75. As compared to without TMD building for the ground motion. The variation of 44.84 % decrease in lateral displacement compared with without TMD is seen for mass ratio Md/M 0.25, 33.90% for mass ratio Md/M 0.5 and 20.85% for mass ratio Md/M 0.75

Table 4.9 Values of Lateral Displacement (30 storey)

COMPARISION OF LATERAL DISPLACEMENT FOR 30 STOREY BUILDING				
lateral displacement at top of a storey along X(mm)	Without TMD	M_d/M=0.25	M_d/M=0.50	M_d/M=0.75
	56.493	37.241	47.012	51.989

From the table it can be concluded that for 30 storey building there is a substantial increase in lateral displacement during the period of strong shaking in the case of Md/M 0.25, 0.5 and 0.75. As compared to without TMD building for the ground motion. The variation of 34.08% in compaed with without TMD is seen in base shear for mass ratio Md/M 0.25, 16.78% for mass ratio Md/M 0.5 and 7.97% for mass ratio Md/M 0.75.

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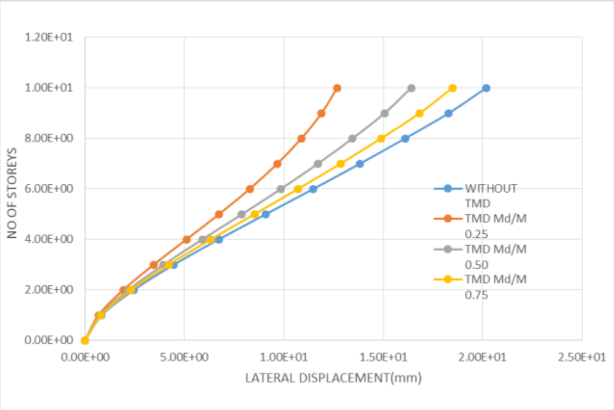

Fig 4.8 Variation of Lateral Displacement of a 10 storey building with and without TMD under different Mass Ratios.

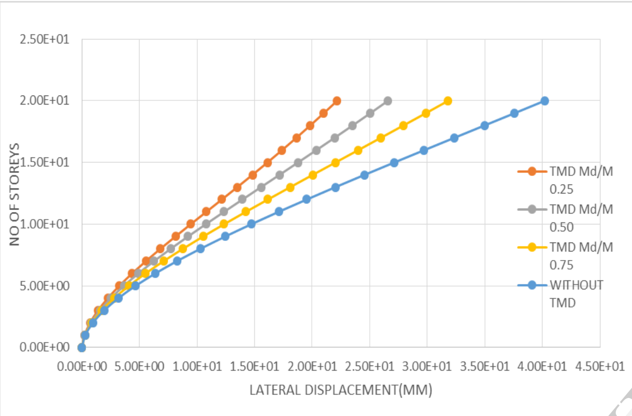

Fig 4.9 Variation of Lateral Displacement of a 20 storey building with and without TMD under different Mass Ratios.

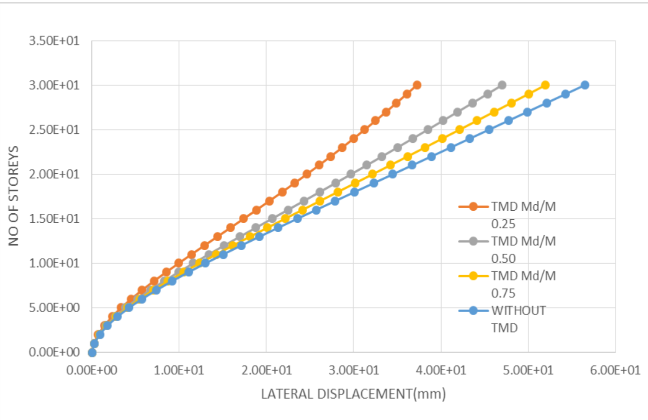

Fig 4.10 Variation of Lateral Displacement of a 30 storey building with and without TMD under different Mass Ratios.

The figure 4.8, 4.9 and 4.10 shows that there is a decrease in lateral displacement with the implementation of TMD under mass ratio 0.25, 0.5 and 0.75 for 10 20 and 30 storey of a RC building. So it is observed that the for mass ratio 0.25 there is least decreases in lateral displacement as compared to that of an mass ratio 0.5 and 0.75.

5.2.2 VARIATION IN ROOF DISPLACEMENT:

Variation in roof displacement due to earthquake motions for various building frames with different mass ratios for ten, twenty and thirty floors has been studied. The time history of roof displacement for various building models with and without TMD are plotted in fig 4.11, 4.12 and 4.13.

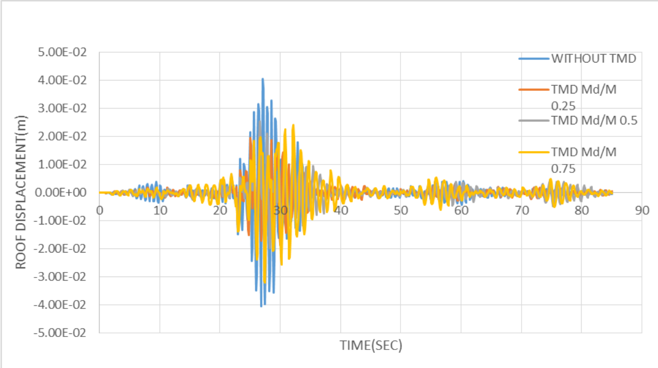

Fig 4.11 Variation of Roof Displacement of a 10 storey building with and without TMD under different Mass Ratios.

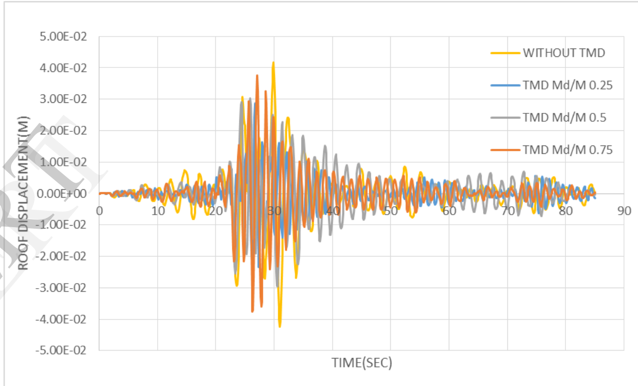

Fig 4.12 Variation of Roof Displacement of a 20 storey building with and without TMD under different Mass Ratios.

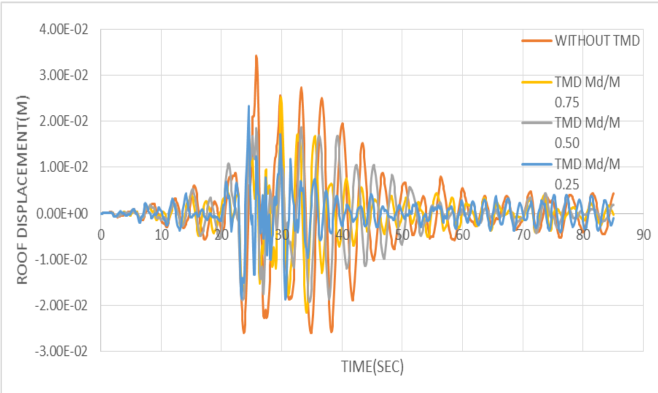

Fig 4.13Variation of Roof Displacement of a 30 storey building with and without TMD under different Mass Ratios.

The figure 4.11, 4.12 and 4.13 shows that there is a decrease in Roof displacement with the implementation of TMD under mass ratio 0.25, 0.5 and 0.75 for 10, 20 and 30 storey of a RC building.it is also observed that for mass ratio 0.25 there is least decreases in Roof displacement as compared to that of an mass ratio 0.5 and 0.75.

5 CONCLUSION

The thesis attempts to study the ability of TMD to reduce earthquake induced structural vibration. Numerical simulation has been performed to compare the multi storey RC structure response with effect of variation of mass ratio of TMD and variation of H/B (1 2 and 3) ratio of multi storey RC structure.

Following conclusions were drawn from the present study:

1. Natural period of a building increases with increase in mass ratio for Md/M=0.75.
2. Base shear increases with the increase in no of stories and there is decrease in base shear for mass ratio Md/M 0.25.
3. Lateral displacement of a multistory RC building increases with increase in no. of storey and after the implementation of TMD there is substantial decrease in lateral displacement under mass ratio Md/M=0.25.
4. TMD are much more effective to reduce roof displacement when mass ratio is less Md/M=0.25.
5. The TMD with mass ratio 0.25 is most effective in controlling the various parameters in a building when compared with mass raio 0.5 and 0.75 in the present study.

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